<table>
<tr><td>Page,
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Date of action</td><td>Corrigendum/Modification</td><td>New text</td></tr>
<tr><td>(August 2008)</td><td>Information</td><td>Nominal valus are in DIN 1054 and subsequently in EAU denominated as Design values. Nominal values are in the meaning of DIN EN 1990, values, which are not determined satistically, but based on experience or on physical conditions (DIN EN 1990, Ziff. 1.5.2.22). in DIN EN 1990 nominal values are handeled as characteristisc values, that means the are used in the the design procedures corrected by partial safety factors. The practice in EAU (but also in DIN 1054) to set nominal values = design values is herewith withdrawn.</td></tr>
<tr><td>p. VII,
preface
(Dec. 2005)</td><td>Corrigendum</td><td>„The incorporation of the partial safety factor concept of DIN 1054 called for a fundamental reappraisal of the methods of calculation and design for sheet piling structures contained in sections 8.2 to 8.4 and the methods of calculation for ~~sheet piles~~ dolphins contained in section 13."</td></tr>
<tr><td>p. 5,
section 0.2.2.4.1
(July 2007)</td><td>Amendment :
Adjustment of the partial safety factor to the 2nd correction of DIN 1054 (April 2007).</td><td>
<table>
<tr><th rowspan="2">Action or action effect</th><th rowspan="2">Symbol</th><th colspan="3">Loading case</th></tr>
<tr><th>LC 1</th><th>LC 2</th><th>LC 3</th></tr>
<tr><td colspan="5">LS 1A: limit state of loss of support safety</td></tr>
<tr><td>...</td><td>...</td><td>...</td><td>...</td><td>...</td></tr>
<tr><td>Favourable permanent actions (dead load)</td><td>$\gamma_{G,stb}$</td><td>~~0.90~~ 0.95</td><td>~~0.90~~ 0.95</td><td>0.95</td></tr>
<tr><td>Unfavourable permanent actions (uplift/buoyancy)</td><td>$\gamma_{G,dst}$</td><td>~~1.00~~ 1.05</td><td>~~1.00~~ 1.05</td><td>1.00</td></tr>
<tr><td>Flow force in favourable subsoil</td><td>γ_H</td><td>1.35</td><td>1.30</td><td>1.20</td></tr>
<tr><td>Flow force in unfavourable subsoil</td><td>γ_H</td><td>1.80</td><td>1.60</td><td>1.35</td></tr>
<tr><td>Unfavourable variable actions</td><td>$\gamma_{Q,dst}$</td><td>1.50</td><td>1.30</td><td>1.00</td></tr>
<tr><td colspan="5">LS 1B: limit state of failure of structures and components</td></tr>
</table>
Table R 0-1</td></tr>
<tr><td>p. 5,
section 0.2.2.4.1
(August 2009)</td><td>Information</td><td>The 4th correction of DIN 1054 contains new partial safety factors for LS 1B, LC 3. These lead to bigger dimensions in the design of waterfront structures. As neither experience nor comparative calculations confirm this, the committee recommends the the use of the old factors γ_G=1.0 and γ_Q=1.0.
For the design of anchors however, the partial safety factors of the 4th correction of DIN 1054:2005 should be used for LS 1B, LC 3 (γ_G=1.10 and γ_Q=1.10).</td></tr>
</table>

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p. 11ff, R 9, section 1.1 (Dec. 2005)	Information	From conversations with colleagues, on the occasion of the HTG-workshops, it became clear, that there is an unsureness in view of the definition of the characteristic soil parameter, due to the partly contradictory use of this term in the past. The committee therefore declares: • According to DIN 4020 (March 03) and DIN 1054 (Jan. 05), characteristic soil parameters are cautiously estimated averages of testvalues (laboratory- and field experiments), therefore safe sided values of the statistic average. This is the meaning they have in the recommendations of EAU 2004, as well. • The previous reductions tan φ'/1.1 or c'/1.3 according to E 96, EAU 1990, were components of the safety concept of the EAU at that time, they are different from the definiton of the design values in terms of DIN 1055-2 (1976)! • As long as, in its annotations, DIN 1055-2 (1976) refers to E 96, EAU 1990, the 10% fractile mentioned there is to be considered. DIN 1055-2 (1976) only requires an „adequate" discount from the average. • According to annex F, DIN 1054 (Jan. 05), the calculating values from DIN 1055-2 (1976) are characteristic values. They are not (see above) identical with the calculating values in the sense of E 96 of EAU 1990!
p. 11, R 9, section 1.1.1 (Dec. 2007)	Modification Modification and complement of the 1st paragraph of section 1.1.1	„The soil properties given in table R 9-1 are ~~cautious mean~~ empirical values for a larger area of soil. They may be used as characteristic values in the meaning of DIN 1054, which is why they are given the index k. Without verification, only the values for low strength in the table can be assumed for natural sands. Medium strength can only be expected after compaction, apart from geologically older sediments. Without verification, the values for soft consistency apply for cohesive soils. The empirical values for the shear parameter of the undrained soil $c_{u,k}$ (column 9) have to be chosen within the given bandwidth, proportional to the geostatic surcharge in such a way, that they correspond with the shear parameters of the drained soil (column 7 and 8)"
p.12;13 R 9, section 1.1,2, Table 9-1	Corrigendum	In Table 9-1, column 4, rows 14 to 18, the consistencies are firm (instead of safe), stiff (instead of stift) and very stiff (instead of semi-firm). In row 19 the consistancies are very soft, firm(instead of soft) and stiff (instead of stift). In row 20 the consistencies are very soft, firm (instead if soft), stiff (instead of stift) and very stiff (instead of semi firm). In row 21 the consistencies are very soft and firm.

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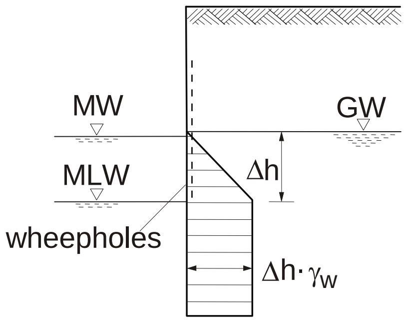
<tr><td>p. 40ff,
R 130,
section 2.6.1
and 2.6.2
(July 2007)</td><td>Information</td><td>The committee indicates that the recommendations for the determination of the active earth pressure in saturated, non- or partially consolidated, soft cohesive soils given in R 130, no. 2.6.1 and 2.6.2 do only lead to identical results, if the cohesive strength c_u of the undrained soil in 2.6.1 is equivalent to the actual cohesive strength at the point of time of the loading. Therefore as a general rule, c_u has to be set variable with depth. Furthermore, the committee emphasizes, that, as a general rule, the earth pressure has to be determined with effective stresses. However, especially for soft cohesive soils, the undrained shear strength determined in situ will often be more reliable than the effective shear parameters in the laboratory.</td></tr>
<tr><td>p. 79,
R 19,
section 4.2
(Dec. 2007)</td><td>Corrigendum:
Adjustment of the drawn description of MW in relation to GW for minor water level fluctuations.</td><td>
<table>
<tr><th colspan="5">Non-tidal area</th></tr>
<tr><td>Situation</td><td>Figure</td><td colspan="3">Load causes as per R 18</td></tr>
<tr><td></td><td></td><td>1</td><td>2</td><td>3</td></tr>
<tr><td>1
Minor water level fluctuations
(h < 0,50 m) with weepholes or permeable soil and structure</td><td>
</td><td>Δh = 0,50 m</td><td>Δh = 0,50 m</td><td></td></tr>
</table>

Figure R 19-1 Situation 1</td></tr>
<tr><td>p. 80,
R 19,
section 4.2
(Dec. 2006)</td><td>Corrigendum:
Table is valid for tidal areas</td><td>
<table>
<tr><th colspan="5">Tidal area</th></tr>
<tr><td>Situation</td><td>Figure</td><td colspan="3">Load cases as per R 18</td></tr>
<tr><td></td><td></td><td>1</td><td>2</td><td>3</td></tr>
</table>

Figure E 19-2</td></tr>
<tr><td>p. 80,
R 19,
section 4.2
(Aug. 2010)</td><td>Inofficial (!)Corrigendum:
Equation in fig. R 19-2</td><td>Fig. R19-2, case 3a: $\Delta h = a + 0.30\ \text{m} + d$</td></tr>
</table>

Page, Re-commendation Chapter, Date of action	Corrigendum/Modification	New text
p. 94, R 113, section 4.7.7.2 (July 2007)	Modification: Complement of the example.	„The following applies in fig. R 113-3 for a high-level stratum with low permeability (case 2a): • Hydraulic head at point D (underside of stratum with low permeability): $h_D = 7{,}00 - 11/15 \cdot 4{,}50$ m $= 3{,}70$ m $(= 2{,}50 + 4/15 \cdot 4{,}50$ m$)$ • Mean hydraulic gradient ~~at base of sheet pile~~ in the stratum with low permeability: $i = \Delta h/\Delta l = (3{,}70 – 2{,}50)/2{,}00 = 0{,}60$ Safety: $i \cdot \gamma_W \cdot \gamma_H \leq \gamma' \cdot \gamma_{G,stb}$ $0{,}6 \cdot 10 \cdot \gamma_H \leq 10 \cdot 0{,}95$ The safety against hydraulic ground heave is guaranteed in the case of favourable subsoil with the partial safety factor $\gamma_H = 1{,}35$, but not in the case of unfavourable subsoil with $\gamma_H = 1{,}8$. The following applies in fig. R 113-3 for a low-level stratum with low permeability (case 2b), in which the sheet piling penetrates 1m: • Hydraulic head at point D (corner of failure body under investigation): $h_D = 7{,}00 - 12/15 \cdot 4{,}50$ m $= 3{,}4$ m $(= 2{,}50 + 3/15 \cdot 4{,}50$ m$)$ • Hydraulic head at base of wall: $h_F = 7{,}00 - 9/15 \cdot 4{,}50$ m $= 4{,}30$ m $(= 2{,}50 + 6/15 \cdot 4{,}50$ m$)$ • Characteristic value of flow force in body of soil in groundwater flow with a width of 3.0m and a thickness of 1m: $S'_k = [(3{,}4 – 2{,}5) + (4{,}3 – 2{,}5)]\ /2 \cdot 10 \cdot 3{,}0 = 40{,}50$ kN/m • The body of soil consisting of 2 stratums (with the same unit weight $\gamma_B = 10$ kN/m³) has a submerged weight of $G'_k = 10 \cdot 3{,}0 \cdot 1{,}0 + 10 \cdot 3{,}0 \cdot 6{,}0 = 210{,}00$ kN/m Adequate safety against hydraulic heave, even with an unfavourable subsoil, is guaranteed when using the partial safety factors to DIN 1054 (section 0) for Load Case 1: $S'_k \cdot \gamma_H \leq G'_k \cdot \gamma_{G,\,stb}$ $40.50 \cdot 1.8 < 210 \cdot 0.95$ $72.9 < 199.5$ In fig. R 113-3 a relatively broad failure body was considered, for better presentation. Usually, following the procedure of TERZAGHI-PECK (fig. R 115-2), the width of the failure body is chosen equal to half of the depth of the wall in the stratum with low permeability (here: 0.5m),representing the most unfavourable failure body.“

Page, Re-commendation Chapter, Date of action	Corrigendum/Modification	New text
p. 127ff R 136, section 5.6.4 (July 2007)	Comment:	In R 136, section 5.6.4 the design wave height is the maximum wave height to be considered for the calculation of a structural element. The effects resulting from the design wave height have to be multiplied with the partial safety factors of the decisive load case, resulting in design values of internal forces and moments.
p. 154, R 102, section 5.13.1 (Dec. 2006)	Corrigendum: HNW has to be corrected to HHW.	Bollards must be arranged on and above the port ground level, with their upper surface above HHW (fig. R 102-1). The bollard diameter must be more than 15 cm. If the bollard does not project sufficiently above HHW, slippage of the line must be prevented with a crosspiece.
p. 156, R 102, section 5.13.2 (Dec. 2007)	Modification: Specification of the 1st sentence in the 2nd section.	„~~The characteristic load is to be taken as 200 kN per bollard and 300 kN for their anchorages.~~ The characteristic load of the wall (i.e. sheet-piling, chords, waling, anchors etc.) has to be taken as 200 kN per bollard. For the anchoring parts of the bollards a characteristic load of 300kN has to be taken.“
p. 159 R 84 Section 5.14.3 (Dec. 2008)	Corrigendum: Footnote of Table R 84-1	The footnote of table R 84-1 restricts the area of claw plates to 10 m², since the condition of uniformly distributed stress is not realistic for larger claw plates. With respect to the structure generally the pressure acting on the structure has to be considered in the design.

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<tr><td>p. 165
R 177
Section 5.15.5
(Dec. 2008)</td><td>Modification:
The text of 5.15.5 is replaced</td><td>
5.15.5 Vertical loads with a rising or falling water level

On structures and piles immersed in ice vertical extra loads act when the water level rises or falls. In case of rising water level water pressure acts beneath of the ice crust as vertical upward force, in case of falling water level the mass forces of the ice crust causes a vertical downward force.

Based on experience both the vertical estra loads are resticted by the bending strength of the ice crust. [xx1]. Thus the vertical forces transferred to piles become

$$A_V = \left(0.6 + \frac{0.15D}{h}\right) \cdot 0.4 \cdot \sigma_c h^2$$
where A_V = vertical extra load [kN]
h = thickness of the ice crust [m]
D = diameter of pile[m]
σ_c = compression strength of the ice crust [kN/m²]

The compression strength of the ice crust is not constant over depth, thus in case of rising water level the compression strength in the lower third of the ice crust thickness, in cas of falling water level in the upper third of the ice crust thickness is recommended in the above equation. In case of ice crusts with less than 50 cm thickness the differentiation of the compression strength his not necessary. The compression strength of the ice crest may be determined according to 5.15.3.1.

[xx1] Kohlhase, S., Dede, Ch., Weichbrodt, F., Radomski, J.: Empfehlungen zur Bemessung der Einbindelänge von Holzpfählen im Buhnenbau, Ergebnisse des BMBF-Forschungsvorhabens Buhnenbau, Universität Rostock, 2006
</td></tr>
</table>

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p. 305 R 7 Section 8.1.4.2 (Dec. 2008)	Modification: Addendum to 3rd break of 8.1.4.2	„In the event of clearance and/or loads above of this the stresses must be checked. In case of Z-profiles with a clearance more than 1,50 m and less than 1,80 m this check is fulfilled by experiments according DIN EN 1990, when the bearing capacity is checked by water pressure simulation or when sufficient local experience with existing structures is available. In all other cases horizontal walings may be used as additional structural elements
p. 382 R 20 Section 8.2.6 (Dec. 2008)	Information	Motivated by often asked questions the committee confirms its recommendation that the design of all anchor transition elements has to be done for the full internal bearing capacity of the actually installed anchors
p. 365f, section 8.2.0 (Dec. 2007)	Modification: Complement of section 8.2.0	„DIN EN 1993-5 accepts, among others, also a plastic-plastic design calculation for steel sheet piling. Such a design, which takes advantage of both, yielding of the cross section and plastic system bearing capacity (for statically indeterminate systems) in the ultimate limit state, could, in special cases, also be useful for waterfront structures. Generally, the methods of elastic-elastic or elastic-plastic design are used, as explained in the following. At present, for plastic-plastic design of waterfront structures, there is missing, among others, experience on the distribution of the earth pressure due to redistribution of internal forces at system yield. The classical earth pressure distributions and the earth pressure figures of EAU may not be taken as a basis for plastic-plastic design calculations.“
p. 367f., R 215, section 8.2.0.2 (July 2007)	Modification: Complement of section 8.2.0.2 behind Figure E 215-1.	„If the aforementioned boundary conditions for the use of the reduced partial safety factor $\gamma_{Ep,red}$ are given, then, for the calculation of the design value of the bending moment, the degree of constraint may be set for the sheet pile length, calculated with not reduced partial safety factors. The internal forces and moments, calculated with $\gamma_{Ep,red}$, are decisive for verifications of the sheet piling.“
p. 371, R 77, section 8.2.2.1 (July 2007)	Modification	„The distances H_E and a as well as the non-redistributed earth pressure distribution $e_{a,k}$ as a result of the dead load of the soil, plus cohesion if necessary, and widespread surcharges on the ground up to 10 kN/m² are defined in figs. R 77-1 and R 77-2. Other surcharges may not be redistributed.”
p. 376 E 4, section 8.2.4 (August 2009)	Modification: R4 ist replaced	The committee has published a new Recommendation 4 in 2009 (Bautechnik) which will replace the old one.

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p. 377, R 4, section 8.2.4.3 (July 2007)	Modification: Complement of section (2).	„(2) Varying vertical force action effects $V_{Q,k}$ Vertical components $V_{Q,k}$ of the action effects due to varying actions Q may not be entered into the equilibrium condition when they have beneficial effects, i.e. ~~do not cause any significant soil support components $B_{v,k}$~~ if they don't contribute considerably to the stresses of the soil support, otherwise see 8.2.4.5. This applies, on the one hand, to action effects that occur directly at the top of the wall, e.g. the water-side support reaction $F_{Qv,k}$ of the superstructure as a result of the actions due to cranes and stacked loads; on the other hand, this also applies to the vertical component $A_{Qv,k}$ of those anchor force components that occur as a result of horizontal, varying actions in the region of the top of a wall or above the anchor position, e.g. • Lateral crane impacts and storm fastenings, • Line pull forces, and • Active earth pressure due to varying actions on the wall section above the anchor."
p. 379, R 4, section 8.2.4.5 (July 2007)	Modification: Cancellation of the last sentence in the 1st section.	„If at the same time another relative displacement between subsoil and sheet pile wall occurs, different to the one that causes the normal formation of the sliding bodies for active and passive earth pressure, this can lead to the need to assume a different angle of inclination. ~~This must be taken into account in the analyses according to sections 8.2.4.3 and 8.2.4.6.~~"
p. 379, R 4, section 8.2.4.5 (July 2007)	Modification: In the recital after the first paragraph, the 3rd article gets modificated.	„… • ~~If relative displacements between subsoil and sheet pile wall are to be expected owing to the subsoil properties around the base of the wall such that the earth pressure angle $\delta_{a,k}$ can have negative value, this must be specified within the scope of the subsoil investigation.~~ A reduction in the angle of inclination $\delta_{a,k}$ leads to an increase in the earth pressure force, which has to be considered when determining the effects on the wall (internal forces and moments) for the verifications in ultimate and serviceability limit states. "
p. 380, R 4, section 8.2.4.5 (July 2007)	Modification: Modifiaction of the last sentence of section 8.2.4.5.	„The corresponding effects should be taken into account in ~~the~~ all analyses ~~to sections 8.2.4.3 and 8.2.4.6~~"

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p. 381, R 4, section 8.2.4.6 (Dec. 2006)	Corrigendum: Specification of the toe resistance to the characteristic value.	„*(4) Characteristic values of part-resistances R_{1i} for LS 1B* The geometrical and constructional conditions according to R 33, section 8.2.11.2, must be satisfied in order to use the toe resistance force $R_{1b,k}$. The axial sheet pile wall displacement required to mobilise a toe resistance force $R_{1b,k}$ is greater than that required to mobilise a skin resistance force $R_{1s,k}$."
p. 382 R 20 Section 8.2.6 (Dec. 2008)	Information	Motivated by often asked questions the committee confirms its recommendation that the design of all anchor transition elements has to be done for the full internal bearing capacity of the actually installed anchors
p. 393ff, R 33, section 8.2.11.2 (July 2007)	Indication	The committee indicates, that in the appendix A 10 of the EAB (2006) empirical values, for skin friction and point resistance of sheet pilings, are communicated. The committee for waterfront structures affirms the using of these values for sheet piling structures in the domain of EAU, as well.
p. 384, R 20, section 8.2.6.3	Modification: According to new DIN EN 1993-5 no proof against the failure of an anchor is needed. The reference to DIN EN 1993-5 is therefore not necessary anymore.	~~The loading case "failure of an anchor" called for in DIN EN 1993-5, section 6.2.2, does not need to be considered for tie rods [...] due to additional stresses caused during installation.~~ It is not necessary to consider "failure of an anchor" for tie rods, because the reduced notch factor k_t^* is used, the anchors' connection is designed for full internal bearing capacity and the tie rods therefore have sufficient load bearing reserves to avoid failures.
p.392, R 33, section 8.2.11 (Dec. 2009)	Information	Replacing R 4 with a completely new version effects R 33. The committee indicates that recommendations concerning vertical loading in R 33 are replaced by the new R 4.
p. 408, R 100, section 8.3.1.3 (Aug. 2008)	Modification: Replacement of Fig. R 100-5	See German correction, p. 426
p. 570, R 189, section 12.5.2 (Dec. 2005)	Amendment	„With regard to mechanical and hydraulic filter efficiency, installation stresses such as tensile and punching forces, and durability with respect to friction stresses with unbonded cover layers, geotextile filters for slope and bottom protection can be designed in accordance with rule stated in [100], [128] and [165]. **[100]** and [165] contain design rules for dynamic loads based on experience with unsteady hydraulic loads."

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New recommendation R 217 (Dec. 2008)	New recommendation R 217	The committee has published a new recommendation R 217, Vertical wave loads (wave slamming) published (in German) in the “Technischer Jahresbericht 2008, Teil 2 (in German)”